VOLUME 86 NO. PO5
OCTOBER 1960
PART 1

JOURNAL of the

Power

Division

PROCEEDINGS OF THE

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OF CIVIL ENGINEERS

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The index for 1959 was published as ASCE Publication 1960-10 (list price $2.00); indexes for previous years are also available.

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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 5, October, 1960.

ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Discussion by J. B. Cooke

Addendum to Closure by Torald Mundal

J. B. Cooke,[1] F. ASCE.—In the conclusion to his paper, Mr. Mundal mentioned two points that the writer would like to emphasize: (a) that rockfill dams can be safely supported on consolidated stream bed material; and (b) that rock not suitable for dumped rock may be utilized by compacting it in layers. Both of these features have been adopted at Brownlee Dam and at Cougar Dam as discussed in this Symposium. Both features are being incorporated in more rockfill dams as the technology advances rapidly and rockfill dams are more frequently adopted. Two dams adopting these economical features are Goschenen Dam in Switzerland, and Messaure Dam in Sweden; both of these dams are now (1960) in an advanced stage of construction. The writer will briefly present data on these two interesting dams using recent information provided by Albert Luchinger of Locher & Cie, Switzerland, and by Tore Nilsson of the Swedish State Power Board. A third major dam is Furnas Dam in Brazil; it is hoped that Mr. Mundal, whose organization is designing the Furnas project, will present data on the Furnas Dam in the Addendum to his Closure.

This discussion also comments on Brownlee settlement and on the definition of rockfill dams.

Goschenen (Goschenenalp) Dam—Switzerland.—Goschenen Dam is a thin central core rockfill dam of 155 m (510 ft) height and 540 m (1770 ft) crest length. The dam contains 9,000,000 cu m (11,700,000 cu yd) of material and creates a 60,000 acre-ft reservoir. Fig. 1 shows the cross section of the dam. References provide information on composition of the core,[2] on shear strength and test fills of the shell rockfill material,[3,4] and on the construction.[5] Construction started in 1955, but due to a 5-month summer construction season, the dam is only now (1960) nearing completion; the whole watershed in this portion of the Swiss Alps is subject to avalanches. The rockfill design was

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[a] August, 1958, by Torald Mundal.

[1] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

[2] "Goeschenalp Rockfill Dam Project, Switzerland," by W. Eggenberger, Proceedings, 3d Internatl. Conf. on Soil Mechanics and Foundations Engrg. Vol. 3, 1953, p. 296.

[3] "Shear Strength of the Materials for the Supporting Shell of the Goeschenenalp Dam, Switzerland," by J. Zeller and R. Wullimann, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957, p. 399.

[4] "Test Fill With Coarse Shell Material at Goschenenalp Dam, Switzerland," by J. Zeller and H. Zeindler, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957, p. 405.

[5] "Die Baulichen Analgen Des Kraftwerke Goshenen," ("The Civil Construction Features of the Goshenen Power Project"), by Rudolf Pfister, Die Wirtschaft, July, 1959, p. 35.

estimated to cost one-half as much as a concrete gravity dam.[5] The downstream slope with 1.45: 1 slopes and benches averages 1.65: 1 for the top 70.5 m (232 ft) height of the dam. This is steeper than has previously been used for this type and height of dam.

Foundation.—The foundation for the core was carried to good granite. The depth of excavation was as great as 230 ft, and amounted to 1,300,000 cu yd. Because the granite was particularly jointed, area grouting was resorted to in addition to the main grout curtain. The grout consisted of a mixture of Portland cement and Opalinus clay. The rockfill rests on granite or on well compacted sand-gravel deposits. The ballast layer, 8 in Fig. 1(a), rests on unreliable layers of clay, sand, peat, and organic matter. The settlement of the ballast zone amounted to 40 in. in 2 yr. The vertical drains, 30-in. diameter, gravel-filled holes, and the drainage bed under the possibly impervious zone 8 are to insure drainage during reservoir drawdown.

Impervious Zone.—The impervious zone is thin because it was necessary to manufacture the material from local sand and gravel and imported clay. The clay is Opalinus clay from the Jura mountains and is used in an amount of 875 lb per cu yd. The material is mixed in a specially built plant and its quality, consistency, and plasticity are rigidly controlled.

Rockfill.—The rockfill material is from Talus deposits. It is not processed and contains a high percentage of fines. Rocks larger than 1 cu m are not used in the fill. Grading curves were made of the natural deposits using 20 cu m to 30 cu m (26 cu yd to 39 cu yd) of material for each sample and up to the 1 cu m blocks. The high percentage of fines is illustrated by the following data; 30% smaller than 10 mm size (0.4 in.) and 44% smaller than 100 mm (4.0 in.) size.[4] Another measure of the high percentage of fines is the low percentage of voids in the fill, 17% to 18%, and the dry density of 2.25 tons per cu m (141 lb per cu ft) for the rock of 2.72 specific gravity. The 17% to 18% voids for the fill containing a high percentage of fines compares with 27% to 30% for cleaner rockfill placed in layers and 30% to 32% for larger rockfill dumped in high lifts. The rockfill at Goschenen Dam is dumped wet in layers 2 m thick and not exceeding 3 m-thick, with no sluicing and with no compaction other than by hauling and spreading equipment. The method of placing rock was based on data from nine test fills containing a total of 30,000 cu m of material and located within the dam.[4]

Messaure Dam.—Messaure Dam is a thin central core rockfill dam of 101 m (330 ft) height. The dam is the largest rockfill dam in Sweden, containing 10,500,000 cu m (13,800,000 cu yd); the highest is the 400-ft Trangslet Dam. Messaure Dam is located on the Stora Lule River 7 miles north of the Arctic Circle. Due to a 5-month summer construction season, the project is taking 5 yr to complete; construction is to be finished in 1962. A description of Messaure Dam and three smaller Swedish rockfill dams, Ligga, Harspranget, and Jarkvissle, is available.[5] Fig. 2 shows the main river cross section and the cross section for the two abutments. Messaure Dam is of interest because of its economical use of available unprocessed materials, use of foundation excavation silt-gravel deposits in the embankment, and use of required project excavations for the rockfill. The decision to carry the abutment sections to bedrock was influenced by the economics of using much of the excavated materials in the (B) and (C) zones of the dam.

Core Foundation.—The foundation for the moraine core is carried to sound rock; all protruding rock formations and precipitous faces are removed so as to prevent any irregular settlements and to provide a longitudinal slope of the

dam not exceeding 1:1 with vertical ledges not exceeding 0.5 m high; no foundation slope is to exceed 1:1 slope; and cavities and crevices are thoroughly washed out and filled with concrete and fissured rock covered with concrete to present an even surface for beginning compaction of the core. The minimum grout curtain consists of two parallel lines spaced 2 m (6.6 ft) apart, with holes at 1 m apart in the downstream row and 2 m apart in the upstream row. The work on the downstream row is completed before the upstream row is started. Two stages; the first to 5 m (16.4 ft), and the second 5 m to 12 m (16.4 ft to 33.5 ft) are used. Water washing, water testing, and grouting are thoroughly carried out.

Zones.—The zones of Fig. 2 are as follows:

A. Moraine Core.—The material is obtained from moraine pits that meet specified clay content, grading and permeability limits. Rocks up to 10 cm (4 in.) size are permitted in rolled fill and up to 25 cm (10 in.) in fill placed by the "wet compaction method," but the sizes are subject to change should the amount of rocks warrant it. The "wet compaction method," developed by the Swedish State Power Board for use with moraines, is described by Mr. Nilsson.[5] It involves compaction of the moraine at a water content near the liquid limit of the material. This requires a material with a limited clay content that is, therefore, reasonably permeable. For Messaure, the moraine complies with the following: the clay content, that portion of the material with grain less than 0.006 mm, must not exceed 10% of material less than 5.6 mm; the fine material content, the portion of the material with grain less than 0.076 mm, must amount to at least 20% of material less than 5.6 mm; and the permeability k-value of the moraine must be between 0.1-0.005 cm per hr determined under a normal pressure of 2 kg per sq cm. The moraine as a rule is placed by the "wet compaction method," the compaction being done by a caterpillar size D-8 or heavier. Where rolled fill is used, it is compacted to at least 95% of the maximum degree of compaction obtained with the modified Proctor method.

B. Transition Zone.—The material consists of coarse silt-sand. Material less than 0.06 mm does not exceed 15% of material less than 5.6 mm. The K-value is specified to preferably be 25-50 times greater than the K-value for moraine. Filter requirements to the (a) zone are complied with. Compaction is to at least 96% of maximum compaction obtained with the modified Proctor method.

C. Gravel Fill.—The material is sand or gravel which after specified compaction shall have a friction angle of at least 35°. The material is placed in layers and compacted at optimum water content to 95% modified Proctor compaction.

D. Supporting Fill (Pervious).—The material consists of sand-gravel with sufficient rock that after specified compaction has an angle of friction of about 40° above and 35° below El. 141. Some of the material comes from abutment excavation. The material is placed in layers and compacted at optimum water content to 95% modified Proctor compaction. Where necessary, filter layers are provided between (D) and (B) zones. Rockfill may be used in the (D) zone.

E. Rockfill (Quarried Rock).—The rockfill material is obtained from a spillway channel, the underground powerhouse and the tail-race tunnel. The grading is specified to contain all sizes with a maximum size of 1 m and with the amount of fines not to exceed 20% smaller than 20 cm. The rockfill is

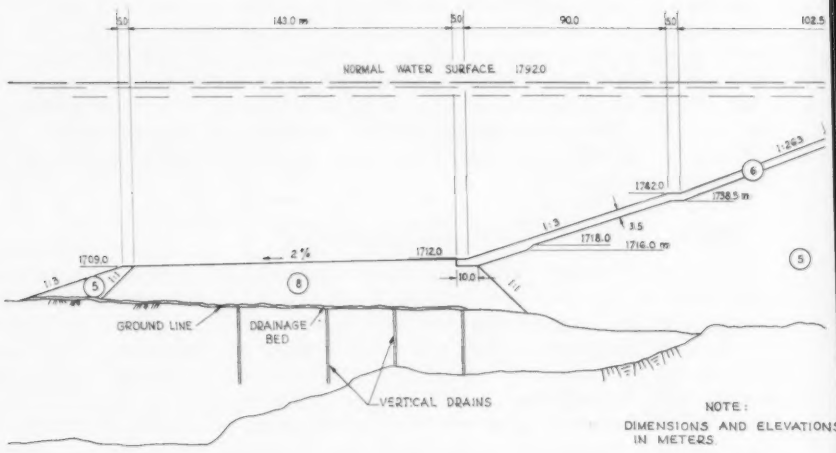


FIG. 1.—CROSS SECTION

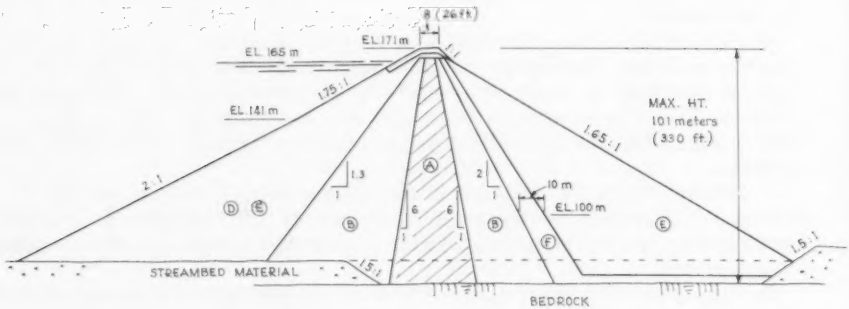
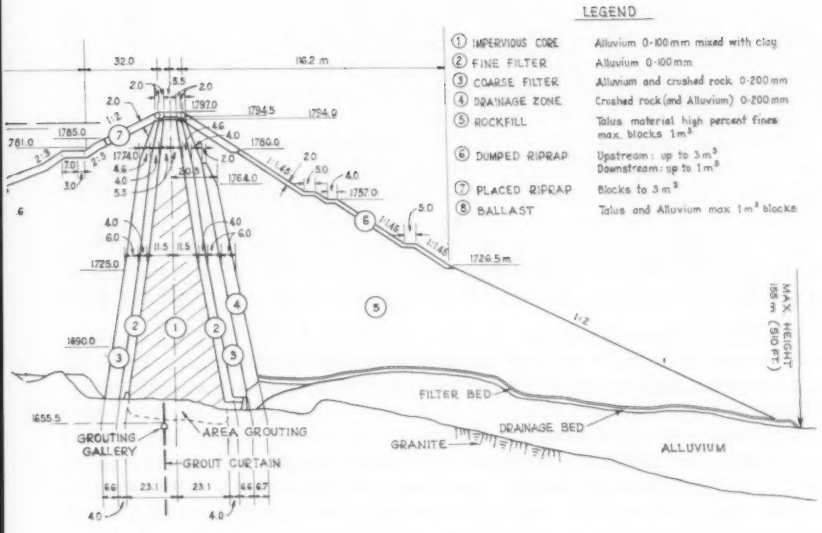
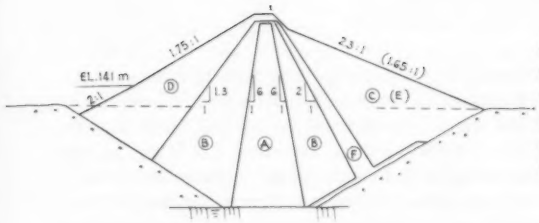


FIG. 2.—CROSS SECTION


LEGEND
① IMPERVIOUS CORE Alluvium 0-100 mm mixed with clay
② FINE FILTER Alluvium 0-100 mm
③ COARSE FILTER Alluvium and crushed rock 0-200 mm
④ DRAINAGE ZONE Crushed rock (and Alluvium) 0-200 mm
⑤ ROCKFILL Talus material high percent fines max. blocks 1 m³
⑥ DUMPED RIPRAP Upstream: up to 3 m³ Downstream: up to 1 m³
⑦ PLACED RIPRAP Blocks to 3 m³
⑧ BALLAST Talus and Alluvium max. 1 m³ blocks
MAX. HEIGHT 155 m (510 FT.)
FILTER BED
DRAINAGE BED
GRANITE
ALLUVIUM
GROUTING GALLERY
AREA GROUTING
GROUT CURTAIN

OF GOSCHENEN DAM



OF MESSAURE DAM

placed in 1 m to 1.5 m layers, compacted by hauling equipment and sluiced with water at a ratio of 2 volumes of water to one of rock. In a 4 m width adjacent to the (B) zone, the maximum rock size is 0.5 m, the maximum layer thickness is 1 m, and placing is not permitted if there is any possibility of freezing. During rock placing under freezing conditions in winter months, sluicing is not used and fines are further limited.

F. Filter and Drainage Zone.—The (F) zone between zones (B) and (E) consists of two zones, common gravel and quarried rock of 25 cm (10 in.) maximum size. Excessive fines are removed where necessary. For the (F) zone between zones (B) and (C) a single (F) zone of natural gravel may be used. Filter grading requirements must be complied with. The permeability K-value is specified to be, preferably, at least 25-50 times as great as the K-value for the (B) material, and that of the (F) zone rock, to be greater than the (F) zone gravel.

The thickness of layers and the number of passes of the various types of compaction equipment, for zones (B), (C), (D), and (F) are determined by compaction tests.

Brownlee Dam.—In his discussion Mr. Condit has presented valuable settlement measurements indicating that Brownlee Dam has performed very well. However, Mr. Condit noted that the crest monuments were installed several months after completion of the rockfill; the water level was already high when the first readings were taken; Mr. Mundal in his closure stated, "Maximum foundation and embankment settlement at Brownlee was 0.546 at Monument F-4 for the recorded period; less than 0.2%." Though the foregoing points are all true, the reader might not get a true picture; the following tabulation is made to show the settlement of point A2 from Figs. II and III of Mr. Condit's discussion:

	Water Depth	Vertical Settlement
(a) About July, 1958 (Complete File)	?	?
(b) 9-25-58 (Begin A, B, and C bench reading)	225	0.46 (B)
(c) 2-25-59 (Begin crest measurements)	235	0.55 (A2)
(d) 9-15-59 (Recorded crest measurements)	275	1.01 plus

The water depths are above the stream bed El. 1800. The 1.05 ft does not include the first months of aging of the completed fill, the filling to 225 ft of the 275 ft full reservoir depth, and the early settlement in the top 40 ft of dam between the ABC bench at El. 2050 and the crest at nominal El. 2090. In the first recorded month for point B, it settled 0.18 ft with little raise in reservoir level. It is probable that in each of the several preceding months the settlement was greater. The data presented are valuable, but it is emphasized that the 0.546 (0.55) ft, for example, does not represent the settlement for the early aging of the completed fill and for the first filling.

It is agreed with Mr. Condit that it is not easy to establish permanent settlement points as soon as desired. However, the lack of complete information at Brownlee, and also at Mud Mountain Dam, suggests that temporary points should be established on such major dams in order that the complete record may be obtained. Even with the loss of a number of temporary points, a complete record can be developed.

Definition of Rockfill Dam.—In considering Goeschenen, Messaure, and Furnas dams to be rockfill dams, the definition must include rockfill compacted in layers. In the paper by John B. Snethlage, et al.,[6] the authors define a rockfill dam: "A rockfill dam is one of which at least half the materials in the maximum cross section is comprised of loose rock placed by dumping." It is possibly not clear whether "loose rock placed by dumping" includes rockfill that is dumped and compacted in layers. Another point in the foregoing definition on which the writer does not agree is the use of "at least half." At least half is, perhaps, too specific.

The writer considers a possible definition to be: "A rockfill dam is one that relies on rockfill, dumped in lifts or compacted in layers, as a major structural element."

TORALD MUNDAL,[7] F. ASCE.—The writer is pleased to present information on Furnas Dam as suggested by Mr. Cooke.

A paper on the overall project consisting of a dam, spillway, power intake, penstocks, and powerhouse has been presented previously.[8] The dam, now under construction, will be a composite fill-type structure containing approximately 9,100,000 cu m (12,000,000 cu yd) of material. It will have a crest length of some 580 m (1,900 ft) and will have a maximum height of about 120 m (395 ft) above the river bottom.

The left abutment of the Furnas site is composed of a finely grained, highly jointed quartzite. The surface of suitable bedrock averages from 5 m to 10 m in depth below natural ground surface over most of the left abutment except at the spillway area where maximum depth is about 35 m (115 ft). The general strike is North 25° West, with the dip 15° to 20° to the Southwest, or downstream and away from the river channel. Bedding planes form joints consisting of very thin layers of clay or fine silts which have been metamorphosed into micaceous planes.

In the main river channel there is a deep, diagonal, fault-formed trough. This has been intruded by a diabase dike that has gouge and fractured rock on both sides.

The right abutment is composed of thick beds of thinly laminated quartzites separated by relatively deep bands of micaceous schists that are fractured and friable near the surface. There has been considerable decomposition and weathering of the schists and a talus slope has formed at this abutment.

Areas containing decomposed schist are present at both abutments. Joints in the quartzites usually are tight and little grouting will be required in most areas. Water testing indicates that little difficulty will be experienced with leakage.

As investigations progressed, it became apparent that a fill-type structure was better suited to site conditions than was a concrete structure. Field explorations indicated that most of the required overburden excavation will be suitable materials for the random fill, and that rock excavation in the intake, spillway, and powerhouse areas can be utilized for the rock shell portion of the dam.

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[6] "Rockfill Dams: Review and Statistics," by John B. Snethlage, F. W. Scheidenhelm and Arthur N. Vanderlip, Proceedings, ASCE, Vol. 84, No. PO 4, August, 1958.

[7] Vice Pres. and Chf. Engr., Internatl. Engr. Co., Inc., San Francisco, Calif.

[8] "Furnas Hydroelectric Project," by James W. Libby, Proceedings, ASCE, Vol. 86, No. PO 2, April, 1960.

Because of the slabby nature of the quartzite fragments to be used for the rock shells, it was considered advisable to make special investigations of the strength characteristics of this material. The United States Bureau of Reclamation in Denver was retained to perform large scale triaxial compression tests on the rockfill using high all around pressures.

Suitable earth-core material is available on the right bank about a mile upstream from the dam axis and on the left bank about 4 miles from the site. Adequate transition zone material is available from spillway overburden excavation. Routine tests of core and transition zone materials have been performed in Brazilian laboratories and have determined the relative stability of the material in the various borrow pits. In addition, test strips have been constructed at the site to correlate field and laboratory compaction results.

The canyon at the Furnas site is 350 m (1,150 ft) wide on the axis at river level. Natural ground extends well above maximum reservoir level on both abutments. Opposed to these favorable conditions the foundation in the river bed is traversed by a deep fault that has been closed with a diabase intrusion.

The section of the dam as shown in Fig. 3 was adopted after both circular arc and sliding wedge analyses showed that it was a structurally stable design.

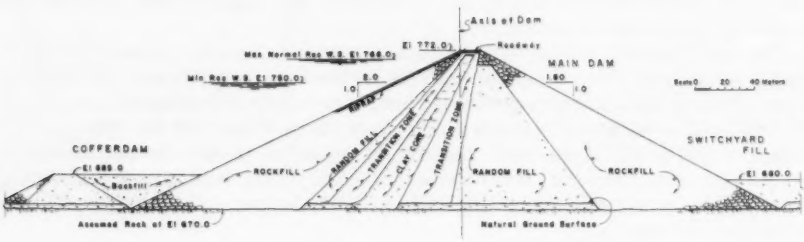

FIG. 3.—FURNAS PROJECT, DAM SECTION

Strength parameters for all materials utilized in the stability analysis were determined from a study of the results of laboratory tests to be as follows:

Zone	c, in psi	ϕ, in degrees
Clay core	14	15
Transition zone	0	38
Random fill	0	38
Rock fill	0	38

The core and transition zones are arranged as a compromise between an inclined core and a central vertical core. This arrangement was found to utilize available materials most effectively. Rockfill will be placed in layers and compacted by crawler-type equipment.

Horizontal curvature of the main embankment has been accentuated to improve abutment contacts and minimize the possibility of cracks developing in the core due to downstream movement (Fig. 4). Because of the steepness of the left abutment, special precautions will be taken to ensure that detrimental effects of differential settlement will not endanger the safety of the core. The upstream and downstream transition zones will be extended along the rock face.

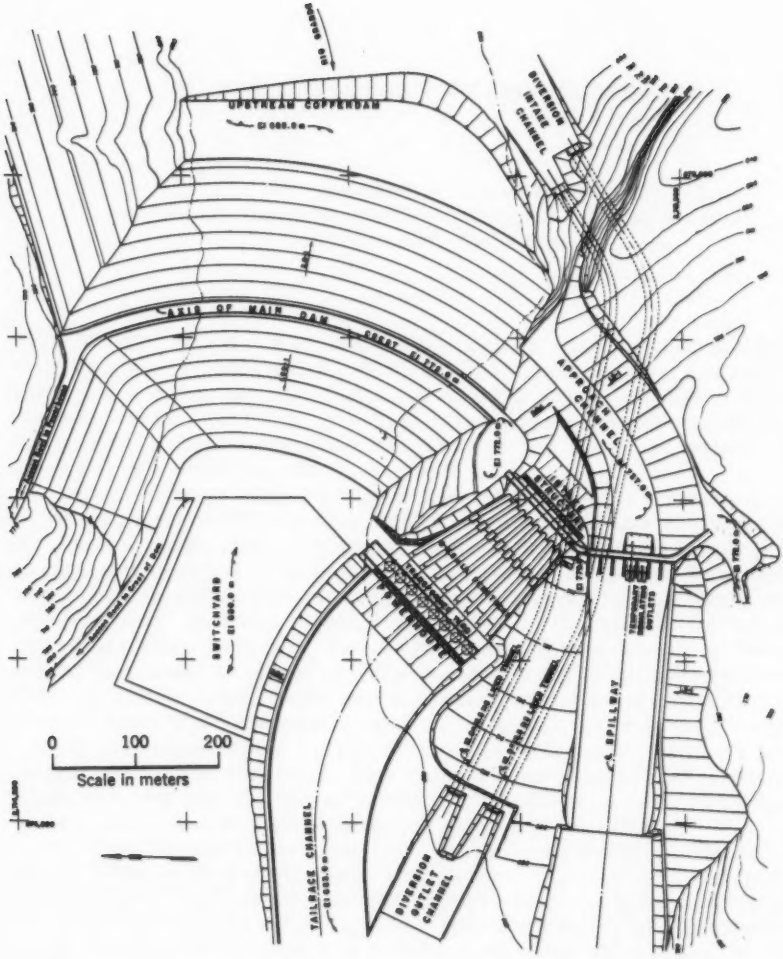

FIG. 4.—GENERAL PLAN OF FURNAS PROJECT

Excavation to suitable foundation on the left abutment is expected to be nominal. The right side will require considerably more excavation to provide a suitable contact area for the clay core against sound-impervious abutment rock. A small amount of gouge and fractured rock excavation in the river channel will expose a suitable foundation.

ROCKFILL DAMS: REVIEW AND STATISTICS[a]

Discussion by G. Robert Koch
Addendum to Closure by John B. Snethlage, F. W. Scheidenhelm, and Arthur N. Vanderlip

G. ROBERT KOCH,[1] A.M. ASCE.—The authors are to be congratulated for their informative paper which constitutes a very useful souce of reference for engineers seeking statistical data on rockfill dams. The list of "Rockfill Dams in Foreign Countries" presented by the authors is most comprehensive. However, it is difficult to make a complete list and the writer submits some additional data which includes some major dams.

The organization with which the writer is associated is presently engaged in the design of the 370 ft high, 10,500,000 cu yd central-core Akosombo rockfill dam in connection with the Volta River Hydroelectric Project for the Government of Ghana. This is the dam listed by the authors in their paper as Volta Dam. In the course of research on other major rockfill dams, certain supplementary statistics to the authors' list of "Rockfill Dams in Foreign Countries" were gathered. These are presented subsequently in the same format and using the same symbols as established by the authors. Because the approximate total volume of embankment of a dam is also a significant characteristic it has been added in the last column of the table given herewith.

It is of interest to note that present planning is to create two of the world's largest man made reservoirs with rockfill dams; 120,000,000 acre-feet at Akosombo, and 105,000,000 acre-feet at High Aswan. Akosombo Dam is not included in the list because it is not yet under construction.

Although the additional dams listed by the writer qualify as rockfill dams on the basis of the authors' proposed definition (that a rockfill dam is one of which at least half the material in the maximum cross section is comprised of loose rock placed by dumping), the writer feels that most engineers would prefer the definition whereby a rockfill dam is one which relies on dumped rock as a major structural element.

The writer gratefully acknowledges the cooperation of the engineers from foreign countries who furnished him some of the data presented in this discussion. The author also wishes to thank his colleagues Lee Worth and J. B. Cooke for supplying him with drawings and information pertaining to a number of the dams.

[a] August, 1958, by John B. Snethlage, F. W. Scheidenhelm, and Arthur N. Vanderlip.

[1] Proj. Engr., Kaiser Engrs. Internatl., Oakland, Calif.

TABLE 1.—SUPPLEMENTARY DATA ON ROCKFILL DAMS IN FOREIGN COUNTRIES

Name of Dam	Type and Height in Feet	Name of River or Creek	Location and Year of Completion	Slopes (Horizontal to 1 Vertical)			Total Volume of Embankment, in 10^6 cu yd
				Upstream Face	Interior	Downstream Face	
BOU-HANIFIA	CAr 175	Oued El Hammam	Algeria 1943	0.8,1.0		1.33	1.0
FURNAS	Er 394	Rio Grande	Brazil u.c.	2.0	0.5	1.80	12.0
GHRIB	CAr 213	Oued Cheliff	Algeria 1932	0.67 + 0 1.0		1.25	0.9
GOSCHENEN	rEr 510	Goeschen-ereuss	Switzerland 1960	3.0, 2.63, 2.0	0.15; 0.15	2.0, 1.4	11.7
HIGH ASWAN	rEr 361	Nile	Egypt u.c.				55
LA JOIE	Wr 285	S. F., Bridge	Canada 1955	1.3, 1.2 1.1, 1.0		1.5	3.7
MARMORERA	rEr 292		Switzerland 1954	3.0, 2.25		2.0	3.5
MESSAURE	rEr 330	Lule	Sweden u.c.	2.0, 1.75	0.17; 0.17	1.65	14.0
PEGO DO ALTAR (SALAZAR)	SWAmr 213	Sado	Portugal 1949	1.25		1.4	0.5
PINZANES	Cmr 180	Tingambato	Mexico 1956	1.2		1.3	0.5
VALSEQUILLO	rEr 269	Atoyac	Mexico 1946				1.0

JOHN B. SNETHLAGE,[2] F. ASCE, F. W. SCHEIDENHELM,[3] and ARTHUR N. VANDERLIP,[4] F. ASCE.—As explained in the original paper, the writers sought to bring, with their definition of rockfill dams, some order into the confusing nomenclature applicable to dams, which, of necessity, must contain, besides rockfill, a second material to form the water barrier.

For example, a dam consisting of 65% earth and 35% rockfill should not be termed a rockfill dam, for no one would deny that the 35% portion is not "a major structural element" of that dam on which reliance is placed. Mr. Koch's proposal is, therefore, surprising, even though he claims to interpret the preferences of "most engineers."

Supplemental Listing.—The writers agree with Mr. Koch that "it is difficult to make a complete list" and thank him for his additions. However, attention must be called to a few apparent errors.

Bon-Hanifia and Ghrib Dams.—These two dams are not rockfill dams, neither by Mr. Koch's nor by the writers' definition.[5] The deck of both dams, constructed on a steeply inclined wall of masonry set in cement mortar, consists of two layers of porous concrete separated by about 5 in. of asphaltic concrete.[6,7] Accordingly, the symbol for each of these two masonry dams should be cAcMm, rather than CAr.

Furnas Dam.—A paper[8] by James W. Libby shows a cross section of the Furnas Dam that indicates the height of the dam to be 102 m or about 335 ft.

Goschenen Dam.—From an available proposed cross section of this dam,[9] it is seen that the height of the main body of the dam, resting on gravel and boulders to be left in place, is about 300 ft; the central earth core, going down to rock, is shown with a height of about 395 ft. Perhaps, however, the design has been changed since the initial announcement.

LaJoie Dam.—The steepness of the upstream face, as listed by Mr. Koch, indicates the presence of at least a rubble masonry wall. The symbol could, therefore, be Wmr, rather than Wr.

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[2] Cons. Engr., East Chatham, N. Y.

[3] Cons. Engr., New York, N. Y. (deceased).

[4] Cons. Engr., New York, N. Y.

[5] Engineering News-Record, Vol. 119, 1937, p. 891 and Vol. 120, 1938, p. 751.

[6] "Montgomery Dam—Rock Fill with Asphaltic Concrete Deck," by F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip, Proceedings, ASCE, Vol. 84, No. PO 1, February, 1958.

[7] Bitumen, German Asphalt Assoc., January, 1951.

[8] "Furnas Hydroelectric Project," by James W. Libby, Proceedings, ASCE, Vol. 86, No. PO 2, April, 1960.

[9] Water Power, February, 1955.

ROCKFILL DAMS: DAMS WITH SLOPING EARTH CORES[a]

Discussion by John Lowe, III

Addendum to Closure by James P. Growdon

JOHN LOWE, III,[1] M. ASCE.—The design and construction principles presented by Mr. Growdon, based on his experience with six sloping core rockfill dams, provide valuable information to engineers concerned with this type of dam. The steep slopes used are particularly impressive.

A discussion has been prepared which briefly outlines the design of the Hirfanli Dam in Turkey and the Binga Dam in the Philippines, both of which are of the sloping core rockfill type. In both, rolled rockfill was used immediately downstream of the impervious core instead of the dumped rockfill used in the six dams described by the author. Information on these two dams is given with the idea that it may supplement the valuable data presented on dams with sloping earth cores.

HIRFANLI DAM

The Hirfanli project provides power and flood control benefits. The project consists of a rockfill dam, ungated spillway, vertical tower intake, power tunnels, penstocks, and a powerhouse. The site is located on the Kizil Irmak River, approximately 80 airline kms southeast of Ankara.

The layout of the dam and appurtenant works is shown in Fig. 1. The height of the dam is 80 m and its crest length 364 m. The spillway has a concrete ogee weir with a crest length of 108 m located in a cut in the right abutment. Two short power tunnels pass through the right abutment; portions of these together with a low level extension were used for diversion. Three 46,000 kva generators, each with Francis turbines as prime movers, are housed in the reinforced concrete powerhouse located at the toe of the dam. Provision has been made for future installation of a fourth unit.

The thickness of the alluvial overburden in the riverbed varied from about 2.5 m to 12 m. Generally, sound gabbro bedrock was found below the alluvium, except for one region on the axis of the dam where a 15 m thick zone of weathered material existed below the overburden. A shallow cover of weathered gabbro existed on the abutments and graded into sound gabbro with depth.

Materials available for embankment construction were sound to highly weathered gabbro from spillway excavation, and sandy, clayey silt from borrow in the flood plain upstream of the dam.

The sound gabbro found in the spillway excavation was hard and dense and had a specific gravity exceeding 3.10. This gabbro was used in the downstream

[a] August, 1958, by James P. Growdon.
[1] Assoc. Partner, Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

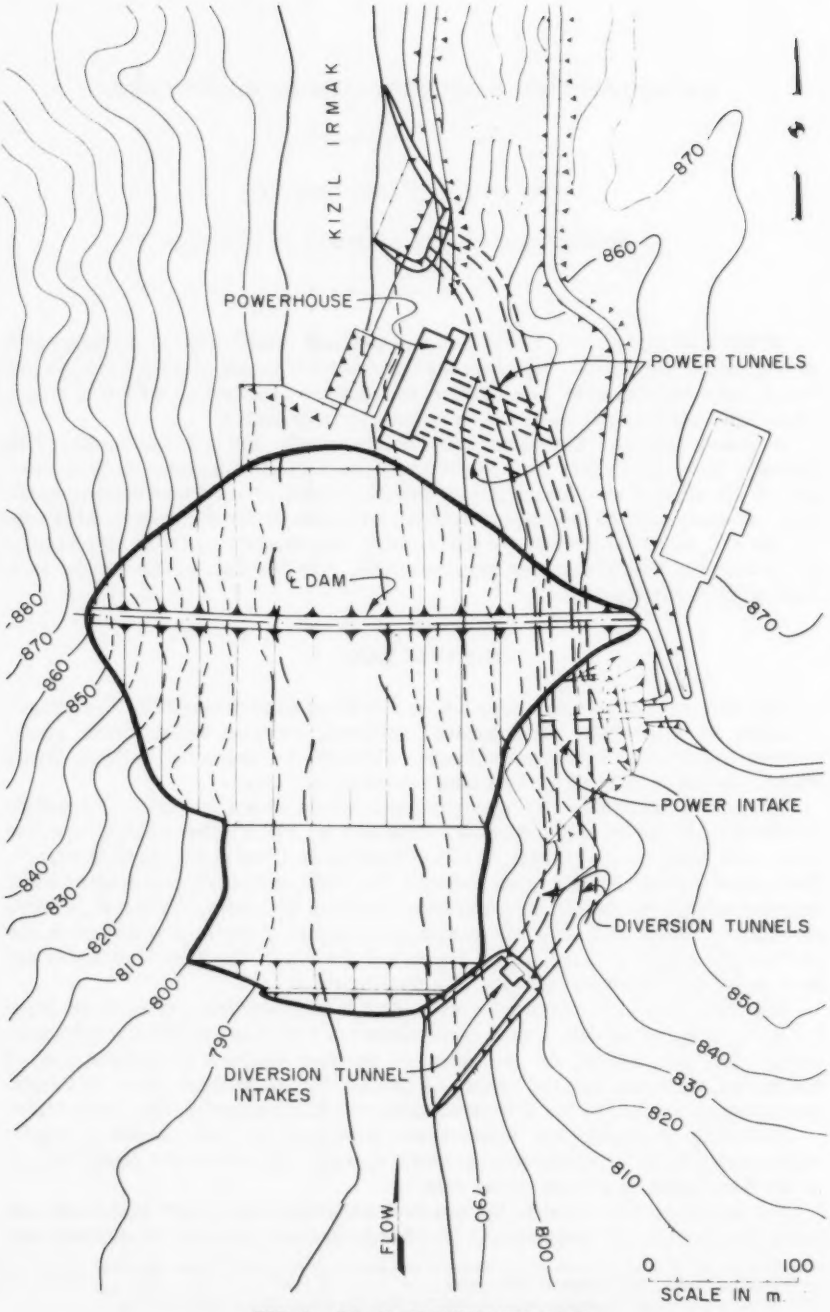

FIG. 1.—PLAN OF HIRFANLI DAM

shell of the dam as sluiced dumped sound rockfill. Much of the excavated gabbro was disintegrated or weathered. The weathered gabbro was used as rolled fill in the center section of the dam immediately downstream of the core. Free draining, coarser, disintegrated gabbro, with little or no fines, was used in the rolled upstream shell of the dam.

Insufficient quantities of natural sand and gravel were available for filters. For this reason sound unweathered gabbro, excavated by selective digging in the spillway cut, was used.

The design of the embankment was influenced by the available quantities of each type of rock and by the order in which these quantities became available during excavation. From preliminary excavation it was estimated that approximately one-third of the total quantity of the spillway excavation would consist of sound rock. In the design of the embankment, therefore, the volume of sound rock was kept to a minimum. As the excavation proceeded, the design was adjusted to the quantities of material of each type actually available, by varying the line of demarcation between the central weathered rock zone and the downstream sound rock zone. The quantities of the structural excavation closely balanced the requirements of the embankment.

During the design stage both a rockfill dam with a vertical impervious core and one with a sloping impervious core were considered. However, the sloping core dam better fitted the topography and foundation conditions. If a central core dam were placed so its core were located at the same place as the sloping core where the depth of alluvium in the river were least, a much larger volume of embankment would be required due to widening of the valley upstream. A cross section of the dam is shown in Fig. 2. The availability of weathered gabbro permitted construction of a large rolled rockfill section immediately downstream of a sloping core. The use of such a section of rolled fill minimizes settlement of the core. It is, therefore, considered by several members of the Board of Consultants and the writer, as a requirement in the design of a sloping core dam in spite of the fact that the author has had good success in constructing sloping core dams without such a feature.

The volume of the materials in the embankment is as follows:

Sound rockfill	866,000 cu m
Weathered gabbro	942,000 cu m
Impervious core	122,000 cu m
Filter material	99,000 cu m
Impervious blanket (cofferdam)	5,000 cu m

The stability of the dam was analyzed for four possible conditions of operation, namely: immediately after construction; static pool at various elevations; effect of earthquake with the reservoir at critical pool elevation; and rapid drawdown from spillway design flood elevation (856.5 m) to the invert elevation of the power intake (818.0 m).

The condition of rapid drawdown was analyzed using a new modification of the Modified Method of Slices.[2] The new method considers the effect of anisotropic consolidation on shear strength. Application of the new method to Hirfanli Dam is described elsewhere.[3] The determination of the shear strength

[2] "Earth Dams," by T. T. Knappen and John Lowe, III, Handbook of Applied Hydraulics, Calvin Davis, ed. (second edition), John Wiley & Sons, 1952.

[3] "Stability of Earth Dams Upon Drawdown," by John Lowe, III and Leslie Karafiath, First Panamerican Conf. on Soil Mechanics and Foundation Engrg., Mexico City, Mexico, September, 1959.

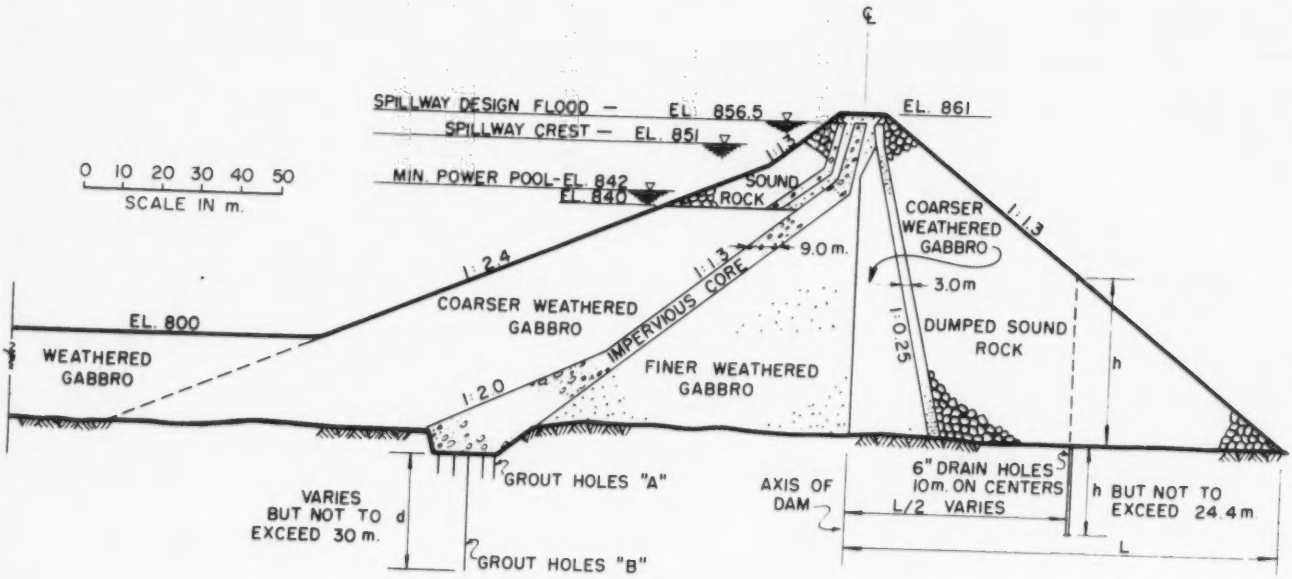

FIG. 2.—CROSS SECTION OF HIRFANLI DAM

for anisotropic consolidation condition for Hirfanli Dam has also been described elsewhere.[4]

Foundation treatment consisted of a grout curtain and drain holes. The depth of grout curtain is equal to approximately one-third the height of the dam. A shallow grout platform was provided under the full width of the core to assure good contact between the core and the curtain. Vertical drain holes were placed at the downstream one-quarter point of the dam to relieve possible water pressures in the downstream section of the foundation.

After construction was completed, a crack developed along the crest between downstream dumped sound rockfill and the filter. The crack is attributed to the dumped rockfill settling more than the rolled rockfill and is not considered serious.

The crack is similar to those mentioned by Mr. Growdon as occurring at Cedar Cliff and Bear Creek dams and attributed to differential settlement. A second possible cause for the cracks mentioned by the author, however, is that the upstream rockfill does not receive hydraulic pressure whereas the earth core behind it does. Thus, the portion of the embankment downstream of the core, shifted downstream due to hydraulic pressure on the core, but the upstream rockfill did not. The curvature of the dams also contributed to the upstream shell not following the horizontal movement of the remainder of the dam.

Acknowledgments.—Hirfanli Dam was designed by Tippetts-Abbett-McCarthy-Stratton for the General Directorate of State Hydraulic Works of the Turkish Government. It was constructed by George Wimpey & Co., Ltd., and the English Electric Co., Ltd., under supervision of the designer and the Turkish Government. Mr. Growdon, Arthur Casagrande, Hibbert Hill, and Friedrich Tolke composed the Board of Consultants.

BINGA DAM

The Binga Project is an integral part of the planned development of the Agno River in Luzon, Republic of the Philippines. It is operated in conjunction with the Ambuklao Reservoir upstream to obtain the optimum output for both power plants. Whereas the Ambuklao Dam has a relatively large storage reservoir, the Binga Dam has a small reservoir limited primarily to regulation of the releases from Ambuklao. The Binga Dam project is primarily a power project and develops a gross head of 154 m with an installed capacity of 100,000 kw. The project consists of a rockfill dam, two diversion tunnels, a gated spillway, a power intake and pressure tunnel, an underground powerhouse, and a tailrace tunnel.

The height of the dam is 103 m with a crest length of 210 m. Two horseshoe shaped diversion tunnels, 7.6 m in diameter, are driven through the left abutment. The spillway is located in a deep side hill cut on the left abutment. It has six tainter gates each 12.5 m wide by 12.0 m high and can pass a maximum flood of 10,000 cms. The power intake is in the right abutment; a pressure tunnel 800 m long and 6 m in diameter leads to the surge tank and steel lined penstock shafts. A 6 m diameter tailrace tunnel, 2,000 m long, returns the water from the underground powerhouse to the river. The general layout of dam and vicinity is shown in Fig. 3.

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[4] "Effect of Anisotropic Consolidation on Undrained Shear Strength of Compacted Clays," by John Lowe, III and Leslie Karafiath, ASCE Research Conf. on Shear Strength of Cohesive Soils, Boulder, Colorado, June, 1960.

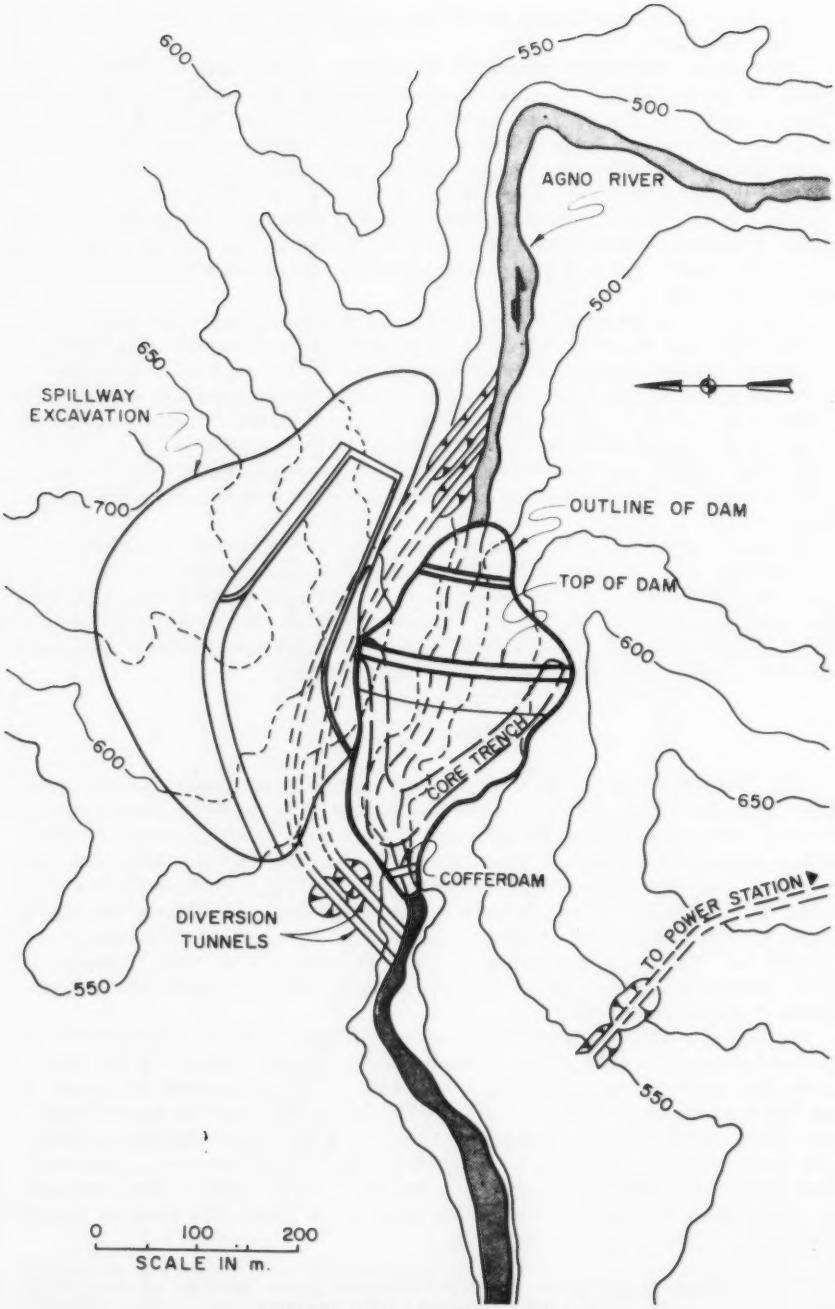

FIG. 3.—PLAN OF BINGA DAM

The dam was built during two construction seasons which run from January through June. During the first construction season it was proposed that a cofferdam be placed, the river diverted through the tunnels, and the dam constructed to El. 569. Four methods of controlling floods between construction seasons were available in this plan: the two diversion tunnels; storage in Binga Reservoir to El. 569; storage above Ambuklao spillway crest; and use of the unfinished Binga Spillway excavated to El. 557.

Originally the formations at the site were lava flows, volcanic ash, and water-deposited sediments laid down in Tertiary time. Subsequently these rocks were metamorphosed with heat, pressure, and folding so that the original structure of the rocks was generally not discernable except on the left abutment where the stratifications of the rocks could be seen dipping upstream. Many fractures and faults resulted from the folding. The metamorphic series was then subjected to the injection of molten diorite and other igneous rocks. These injections appeared as dikes and widely distributed veinlets in the fractures. The faults at the proposed dam site were minor and generally occurred before the injection of the igneous rocks. There was no indication of movement on them.

Weathering of the rocks at the site was generally deep but did not extend below ground water table. Weathering was greater on the right abutment than on the left because a set of minor faults running SE intersected a set running SW in that area. At higher elevations there was present on both abutments residual soil resulting from disintegration and decomposition of the parent rocks. Because of the tropical climate these soils contained a preponderance of iron oxides due to leaching out of silica. Between the residual soil cover and unweathered rock a transition zone of broken soft rock existed. The unweathered rock was hard but brittle and generally broken. It varied from being highly broken rock to occasional relatively sound massive rock at depth.

Only a small amount of sound and massive rock was present. The rock was exposed in the river valley and in much of the left abutment. The upper part of the left abutment and most of the right abutment had only a thin mantle of overburden soil and transitional soft rock overlying the hard, but brittle and broken, bed rock. In the fault areas the rock was much more broken than in between the faults. To minimize grouting problems the impervious core was designed to contact the foundation in between faults.

In the spillway excavation the major material produced was hard, but broken, rock suitable for rockfill. About 80% of the material was less than the 30 cm size and was fairly well graded. It was suitable for placement as rolled rockfill in 45 cm layers provided the plus 30 cm material was raked out. The raked out material was suitable for dumped rockfill. A small percentage of the excavation was in sound relatively massive rock and this rock produced material directly suitable for sluiced dumped rockfill.

A deposit of sand and gravel was found as a terrace near the dam site and was suitable for filters on either side of the impervious core of the dam. The material was well graded and compacted well.

The spillway excavation produced two materials which were suitable for the impervious core, namely the soil cover from the upper elevations and this same soil cover mixed with some of the disintegrated rock that occurred below it. The mixture was the better material since it had a somewhat higher shear strength. Stockpiling of this material would have been necessary, however, and in order to avoid wetting of the stockpiled material during the rainy season, it was decided to borrow the core material directly from an area in the vicinity

of the power intake. The core material borrowed from this area consisted of disintegrated metamorphic rocks.

The scheme of diversion proposed consisted of diverting the river through two horseshoe shaped tunnels and constructing the dam to El. 569 during the first dry season. This height of pool developed sufficient capacity in the two diversion tunnels and in the incompleted spillway to handle a one in thirty year flood. The type of first stage construction which involved minimum volume was a rockfill section with angle of repose slopes upstream and downstream of 1 on 1.4. In order to be stable with pool at El. 569 and upon drawdown after a flood, the rockfill cross-section was constructed in a particular manner. For stability under full pool, the permeability of the upstream portion of the rock had to be less than that of the downstream portion. If this were not so, the force of water seeping through the dam would have caused stability failure downstream. The majority of the rock from the spillway excavation was placed as rolled rockfill and on this basis the upstream third of the first stage construction was made of the finer size rolled rockfill, the middle third of coarser sized rolled rockfill, and the downstream third of large sound rock sizes suitable for dumped rockfill. For stability of the first stage construction upon drawdown of the reservoir, the rolled rockfill for the upstream third was not sufficiently pervious to be free draining upon drawdown. Therefore, upon drawdown, the water in the rolled rockfill would drain at a slow rate and the seepage would be in an upstream direction, causing an unstable condition. To counteract this condition a protective layer of dumped rockfill was proposed for placement at the completion of first stage construction. However, the owner elected to take the risk of a severe flood not occurring and won the risk.

During the second construction season the sloping core was continued on top of the upstream face of the rockfill resulting from the first stage construction and an upstream rock shell was placed to cover the sloping core. Since the rock dips in an upstream direction, according to exposures on the left abutment, the sloping core had the additional advantage of tying in better to the foundation than a vertical core. The horizontal thickness of the impervious core at spillway crest level was 10 m and at valley bottom 30 m. This thickness of core resulted in a maximum hydraulic gradient of 5 for reservoir at normal pool level. A cross section of the dam is shown in Fig. 4.

Filters were placed between the downstream rockfill and the core and between the core and upstream rock shell. This material consisted of sand and gravel from the terrace near the dam site. Construction of the upstream rock shell was accomplished in a manner similar to that used for the downstream rock shell with the finer rolled rock placed adjacent to the filter layer on the upstream side of the impervious core. The filter layer downstream of the core was 3 lanes (7.5 m) wide and the upstream 2 lanes (5.0 m) wide, both up to the crest of the first stage construction.

The slope of the downstream face of the dam was the angle of repose of dumped rock which was 1 on 1.4. The slope of the upstream face below El. 563 was 1 on 2.5. Above El. 563 a center core rockfill dam section with an upstream slope of 1 on 1.4 was used.

The dam was cambered from El. 584 at the abutments to El. 586 at the midpoint of its length. The axis of the dam was curved upstream so that under water load the rockfill downstream of the core tended to wedge between the valley walls due to slight arch action. The curvature was such that the midpoint of the curved axis was 10 m upstream of the chord line drawn between the ends of the dam at crest elevation.

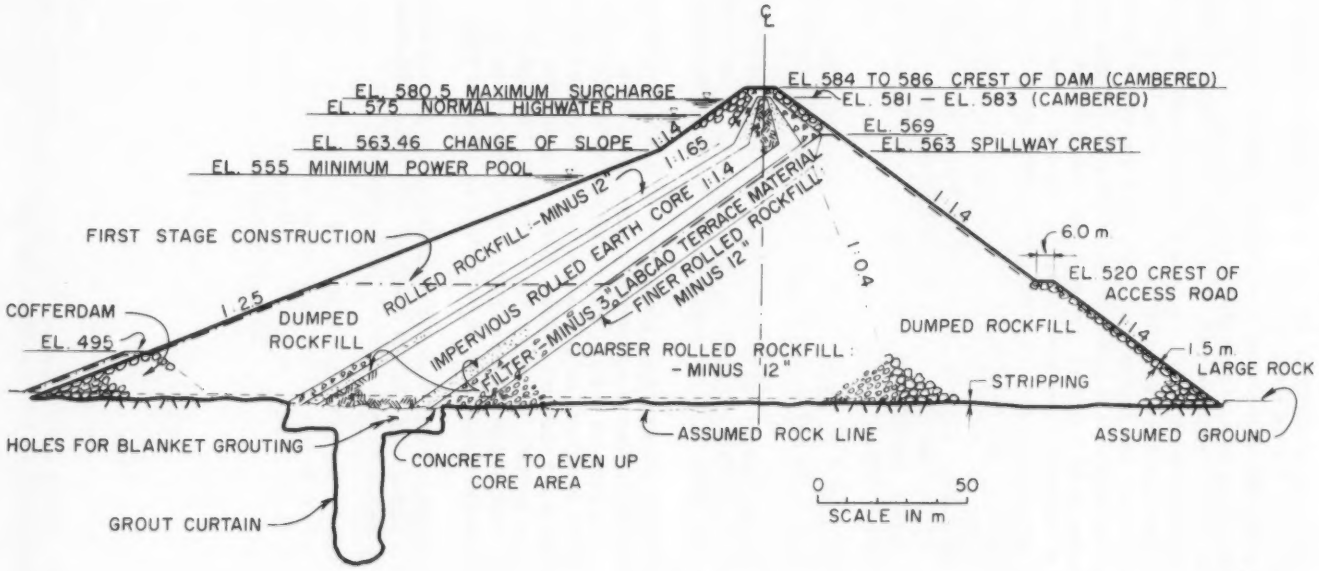

FIG. 4.—CROSS SECTION OF BINGA DAM

The method of stability analysis for the Binga Dam was essentially the same as that performed for the Hirfanli Dam.

Grouting of the foundation rock was required for tying the impervious core into the foundation. There was no zone of relatively unbroken impervious rock to which the grout curtain could be tied, so the minimum depth of grouting in the valley was to a penetration of 1/2 the head developed under the maximum normal reservoir level but not less than 25 m. In order to tie the grout curtain to the impervious core, the foundation of the core was either excavated to groutable rock and an impervious platform formed by grouting this area using 5 m deep grout holes, or a 0.5 m thick concrete mat was provided. Where open seams appeared at the surface they were filled with concrete from the surface. The row of deep grout holes for the grout curtain were grouted to the platform and the core placed tight against the platform.

Acknowledgments.—The owner of the project is the National Power Corporation of the Philippines. Tippetts-Abbett-McCarthy-Stratton together with the Engineering and Development Co. of the Philippines were consultants to the Corporation during design and construction. Construction was by the Philippine Engineering Syndicate Inc.

JAMES P. GROWDON,[5] F. ASCE.—Mr. Lowe's discussion is a brief and adequate description of the design and construction of two sloping earth core rockfill dams.

The optimum in dam design and construction is to build a structure which is safe against all the forces of nature which tend to destroy it, and at the least cost. To accomplish this it is necessary to use correctly materials most readily available.

A dumped rockfill requires sound large rock so that the voids in the fill can absorb the fines which always accompany quarry operations.

The required excavation for the projects described by Mr. Lowe produced some rock suitable for a dumped rockfill and some rock which was unsuitable because of the small sizes and excess fines. The rock unsuitable for dumped rockfill was entirely suitable for a compacted rockfill. The dumped rockfill and the compacted rockfill can be interleaved without the necessity of determining in advance the exact quantities of each class of material which will be available.

Mr. Lowe's discussion illustrates the flexibility of the sloping earth core rockfill type in the hands of a skillful designer. Mr. Lowe's discussion has added materially to the usefulness of the writer's paper.

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[5] Consultant, Aluminum Co. of America, Pittsburgh, Pa.

ROCKFILL DAMS: PERFORMANCE OF MUD MOUNTAIN DAM[a]

Closure by Allen S. Cary

ALLEN S. CARY,[1] A.M. ASCE.—The discussions and questions raised by Messrs. Cooke and Lawton are pertinent and should be completely answerable from the record. Unfortunately, the records are not as complete as the author originally believed. It has come to light that the monuments on the top of the dam along the axis and on the slopes were not set for a period of 70 days after completion of the dam. The statement that the 1950 measurement indicated a total settlement of 2.7 ft is, therefore, incorrect.

Fig. 1 shows a section of the dam along the axis with the measured settlement of the gages at El. 950, 1050, and 1150 ft plotted at an exaggerated scale.

Fig. 2 shows, graphically, the settlement measurements on all gages. Readings taken from 1947 to 1950 indicate practically no movement.

Horizontal movement of monuments on the top of the dam set 70 days after completion along the axis indicate lateral movements by 1949 as follows:

Monument No.	Downstream Component	Upstream Component
8A	0.02 feet	
8B	0.08 feet	
8C	0.11 feet	
8D	0.11 feet	
8E	0.05 feet	
8F		0.20 feet
8G		0.15 feet

The upstream and downstream components of movements are attributed entirely to topography of the canyon walls. Fig. 3 indicates the horizontal component of the downslope movement of monuments set on the slopes.

A great percentage of the rock is hard and brittle showing no evidence of weathering. A small percentage of the tuffaceous andesite has weathered severely, but the total volume of this rock is so small that it can affect the total settlement of rock shells very little.

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[a] August, 1958, by Allen S. Cary.

[1] Geol., Soils Lab., U. S. Engrg. Dept., Univ. of Wash., Seattle, Wash., Formerly, Chf., Foundation and Materials Branch, U. S. Army Engr., Dist. Seattle, Wash.

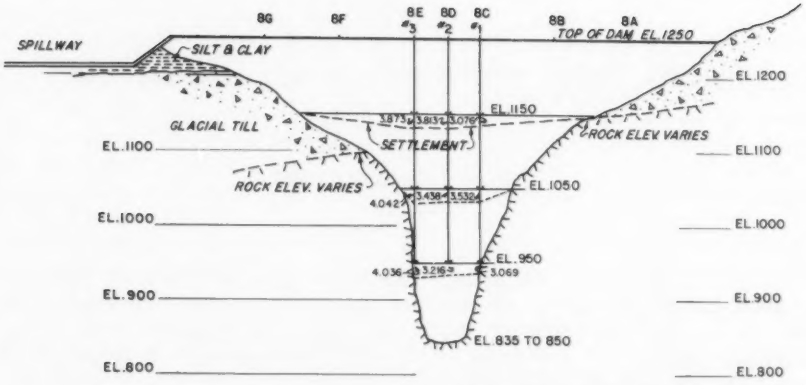

FIG. 1.—SECTION ALONG AXIS OF DAM, LOOKING UPSTREAM

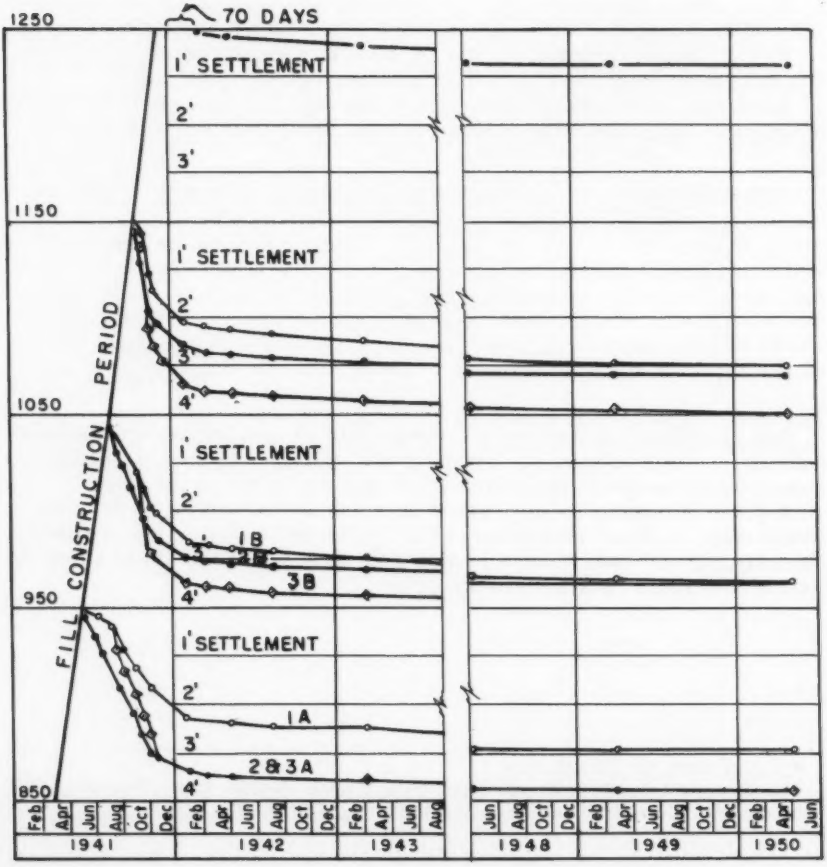

FIG. 2

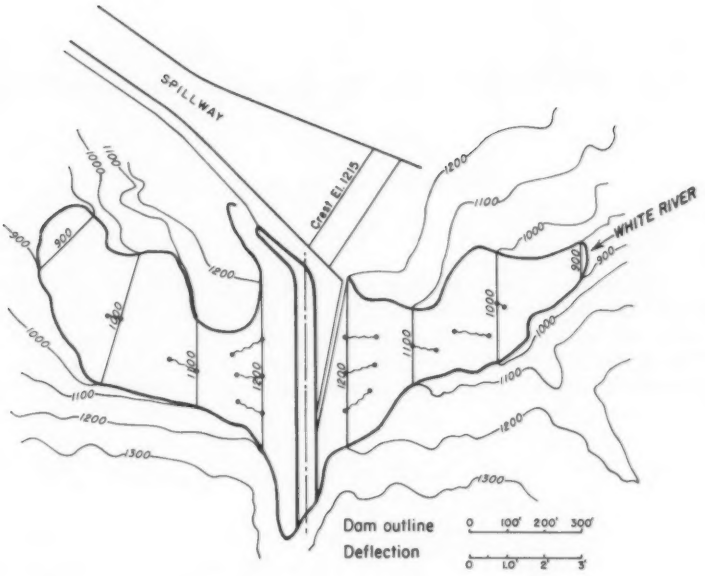

FIG. 3.—LATERAL DEFLECTION OF MONUMENTS TO MAY 1950

ROCKFILL DAMS: DESIGN OF COUGAR CENTRAL CORE DAM[a]

Closure by Paul Thurber

PAUL THURBER,[1] M. ASCE.—The embankment design for Cougar Dam had not been completed at the time the original paper was published in August, 1958. The completed design differs slightly from the design reported in that paper, both in section and in materials. The embankment section is shown in Fig. 1. The modifications, in general, were based on information obtained from

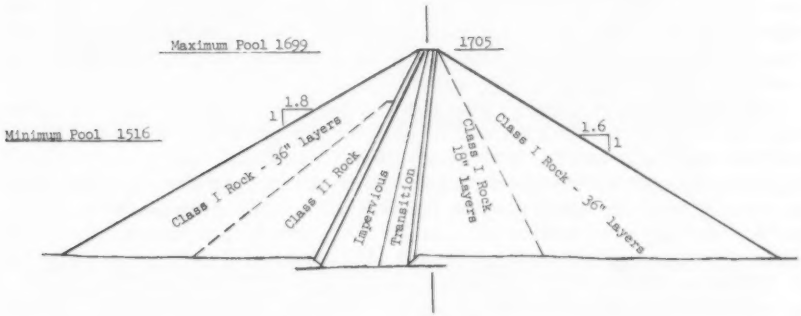

FIG. 1.—EMBANKMENT SECTION

additional investigations of the site and available materials. This closure describes those modifications, discusses the reasons for adopting them, and reports on the progress that has been made in construction of the dam.

The results, obtained from the investigation of the material stripped from the abutments and stockpiled upstream from the site, indicate that this material is more suitable for use in the impervious core than the MH material originally selected. This material is, predominantly, a mixture of basalt talus and sandy silt, with 25% to 55% passing a No. 4 sieve and 10% to 30% passing a No. 200 sieve. The fine fraction is a border-line ML-CL material. Additional explorations have located a source of suitable transition material in an old terrace downstream from the site. The material is a well-graded gravel and sand with 20% to 45% passing a No. 4 sieve and 0% to 15% passing a No. 200 sieve. The fine fraction ranges from non-plastic thru ML to CL material. The same placing procedures will be used for both materials; boulders and rocks larger than 6 in. will be removed, and the materials will be spread in 12-in. layers and compacted with four coverages with a 50-ton rubber-tired roller.

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[a] August, 1958, by Paul Thurber.

[1] Chf., Foundations and Materials Branch, U. S. Army Engr. Dist., Portland, Ore.

The placing moisture content of both materials will be controlled within a narrow range of optimum.

The base of the impervious core has been moved upstream so the contact with the rock foundation will be upstream from the crest of the steep-sided basalt ridge which projects into the valley at the left abutment. Although the core is inclined, the dam is considered to be a central-core design because the inclination is not sufficient to require flattening the lower portion of the upstream slopes for stability. The impervious core is narrower than in the previous design because the transition material is sufficiently impervious to serve as a secondary core. The transition zone also eliminates the need for the two-zone filter in the previous design. The upstream slope of the impervious core will be protected by a layer of transition material, and there will be a layer of rock spalls between the transition zone and the rockfill downstream.

It was learned, from excavation for the relocated forest service road on the left abutment, that the jointing in the basalt formations is such that large rock suitable for placing in high lifts and sluicing as originally planned will be difficult to obtain; the placing procedure for rockfill has been modified accordingly. Rock from the road excavation which was dumped over the left abutment to be used later in construction of the dam was so broken up by the impact that quarrying methods, which drop the rock more than 20 ft, are prohibited in the contract specifications.

The rockfill zones, shown on the embankment section, are designed to provide a section of well-compacted rockfill to support the core and to use the available materials as effectively as possible. Class I rock will be clean, sound fragments of unaltered basalt or andesite with not more than 12% of the fraction passing the 6-in. sieve passing the 1-in. sieve. The rock placed in 18-in. layers will have a maximum size of 12 in. in shortest dimension, and each layer will be compacted by two coverages with a crawler-type tractor weighing not less than 30 tons and two coverages with a 50-ton rubber-tired roller. The rock placed in 36-in. layers will have a maximum size of 24 in. in shortest dimension with not less than 30%, by weight, in pieces larger than 12 in. in shortest dimension, and each layer will be compacted by two coverages with the tractor. If there is a surplus of small-sized rock it may be placed in this zone in 18-in. layers, but each layer will be compacted by two coverages with the tractor. Oversized rock will be placed on the upstream and downstream faces of the dam without compaction. Class II rock will be basalt, andesite, or coarse-grained tuff with up to 25% of weathered pieces permitted, or sandy boulder alluvium from foundation excavation, with a maximum size of 18 in. in shortest dimension and not more than 25% passing a No. 4 sieve. This material will be spread in 24-in. layers and compacted by two coverages with the tractor.

It is believed that compaction of the rockfill downstream from the core will reduce, or eliminate, movements in the dam when the reservoir is filled. This might affect the core, and it is planned to do considerable experimentation with different types of rollers to develop an effective method of achieving this compaction. Plans for a steel impact roller are being developed, and this will be used along with standard rollers of various types.

The contract for construction of Cougar Dam was awarded in June, 1959. The diversion tunnel was constructed under a previous contract, but rock conditions at the upstream entrance required the re-design of the upstream portal, and this structure was included in the dam contract. The contractor was also required to install facilities for trapping salmon downstream from

FIG. 2.—DAMSITE LOOKING UPSTREAM (JUNE 1960) FOUNDATION EXCAVATION IN PROGRESS

FIG. 3.—PENSTOCK TUNNEL (JUNE 1960)

FIG. 4.—EXCAVATION IN BURIED CHANNEL, LOOKING UPSTREAM FROM DOWNSTREAM TOE

FIG. 5.—EXCAVATION IN BURIED CHANNEL, LOOKING UPSTREAM THROUGH CORE AREA

the diversion tunnel, transporting them through the reservoir area, and releasing them in the upper river before diverting the river flow through the tunnel. The tunnel portal and fish facilities were completed in time to divert the river flow at the end of the winter flood season in April, 1960. Excavation for the inlet structure and the penstock and regulating tunnels was essentially complete in June, 1960, and considerable progress had been made in excavating the foundation for the dam and in developing the principal quarry on the right abutment downstream from the spillway. Views of the progress that has been made are shown in Figs. 2 and 3.

Exploratory drilling in the valley floor was interpreted as indicating two fairly extensive but unconnected potholes in the foundation bedrock, but excavation in this area has revealed a buried channel that is continuous for the full length of the foundation. This channel is 60 ft to 80 ft wide in the central portion of the foundation where it has been eroded through a basalt cap into the underlying tuff and siltstone, and widens out upstream and downstream where the basalt is not present. The width near the upstream and downstream toes had not been determined when this was written. Views of the excavation in the buried channel are shown in Figs. 4 and 5.

Additional exploratory drilling in the channel has been limited to avoid interfering with the contractor's operations. From information that has been obtained it is expected that the channel will be around 30 ft deep at the upstream toe and 70 ft to 80 ft deep in the core area and near the axis. Present plans are to excavate the material in the channel to rock in the core and to fill the channel with low-strength concrete for the full width of the core, and to excavate the material in the channel to a depth of approximately 35 ft where the channel is less than 100 ft wide upstream and downstream from the core area. The depth of excavation in other areas will not be decided until more information has been obtained.

The writer agrees with Mr. Cooke's comments on the value and importance of measurements of lateral movements of points on the crest of a rockfill dam. His organization has been a leader in obtaining these data, and the designers of Cougar Dam are indebted to them for information made available during the design.

ERRATA

Journal of the Power Division

Proceedings of the American Society of Civil Engineers

February, 1960

p. 85. For Manuherikia Dam the symbol for the dam type should be CMmr.

p. 86. In line 15 from the bottom change CMr to Cmr.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

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MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

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c. Discussion of several papers, grouped by divisions.

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PART 2

OCTOBER 1960 — 37 NO. PO 5
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE POWER DIVISION OF ASCE

JOURNAL OF THE POWER DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

POWER DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

October, 1960

ASCE SYMPOSIUM ON ROCKFILL DAMS

At the meeting of the ASCE Board of Direction, in Reno in June 1960, it was arranged that the complete Symposium on Rockfill Dams of some eight hundred pages be published in one bound volume as ASCE TRANSACTIONS, Volume 125 (1960) - Part II (Rockfill Dams).

The volume will include all ASCE literature on the subject of rockfill dams from October 1954 to October 1960. It will comprise 25 papers and 75 discussions; the discussions following each paper. This special volume of 1960 ASCE TRANSACTIONS will provide a comprehensive, up-to-date and convenient reference volume on rockfill dams.

ASCE TRANSACTIONS, Volume 125 (1960) - Part II (Rockfill Dams) will be available in about January 1961 and will be priced at $12, making it available to ASCE members at $6 with the usual 50% discount. The $6 price has also been made available to members of USCOLD and ICOLD who place advance orders by October 31, 1960.

To reserve a copy, please write to Executive Secretary, ASCE, 33 West 39th Street, New York 18, New York, with advance payment. This will assure you of a copy at the earliest possible date. For your convenience, an advance order form is provided on the last page.

The Symposium has been planned to assemble design, construction and performance data on most of the world's higher rockfill dams. The presentation of settlement data has been emphasized and detailed data on many high dams has been made available for the first time in the papers and discussions.

For purposes of the Symposium, a rockfill dam was defined to be one that relies on rock, either dumped or compacted in layers, as a major structural element. Included are rockfill dams of the types with (1) impervious face membranes, (2) sloping earth cores, (3) thin central cores, and (4) thick central cores. Some of the notable dams covered in the Symposium and listed by the above types are: (1) Dix River, Salt Springs, Montgomery, Cogswell, San Gabriel, Lower Bear River, Paradela, Wishon, Courtright, Pinzanes, Nozori, and Ishibuchi; (2) Nantahala, Kenney, Brownlee, Bersimis, Dalles Closure, Trangslet, Miboro, Hirfanli, Queens Creek, Cedar Cliss, Bear Creek and Furnas; (3) Ambuklao, Makio, Derbendi Khan, Kajakai, Mud Mountain,

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Goschener, Messaure and Cougar; and (4) Nottely, Watauga, South Holston and Cherry.

Rockfill dams are being increasingly adopted throughout the world and are being constructed to ever-increasing heights. This Symposium will certainly contribute toward improved, more economic and higher rockfill dams of all types.

POWER CONFERENCE OF KANSAS CITY SECTION

The Kansas City Section of the ASCE will be hosts for the local Section Conference on Power to be held at the Hotel Continental in Kansas City, Missouri on November 10-11, 1960.

To help plan adequate accommodations for events at the conference, the Conference Committee requests early notification of plans for attendance. It will be appreciated if those who plan to attend will please advise at their earliest convenience Ray E. Lawrence, Black & Veatch, 1500 Meadow Lake Parkway, Kansas City 14, Missouri.

Registration will be in the Continental Hotel Sixth Floor Lobby during the following hours:

Thursday, November 10, 8:00 AM to 12:00 Noon

Friday, November 11, 8:00 AM to 9:00 AM

The registration fee for members and guests will be $5.00. There will be no charge for ladies, children and students. The fee includes a copy of the published proceedings of the conference. All sessions will be held in the Continental Room on the Sixth Floor.

THURSDAY MORNING, NOVEMBER 10

9:15 AM to 11:45 AM

Presiding: Ray E. Lawrence, Conference Chairman; Partner, Black & Veatch, Consulting Engineers, Kansas City, Missouri.

Welcome: Jack F. Daily, President, Kansas City Section, ASCE; Partner, Lutz & May, Consulting Engineers, Kansas City, Missouri.

Keynote Address - The Power Industry: Where Is It Going?

Walker L. Cisler President, ASME; President, The Detroit Edison Company, Detroit, Michigan.

How Shall We Select the Plant Site?

A. F. Hartung, Member of Firm, Burns & McDonnell Engineering Company.

THURSDAY LUNCHEON - NOVEMBER 10

12:00 Noon
Georgian Room - Fifth Floor

Speaker: R. A. Olson, President, Kansas City Power & Light Company, Kansas City, Missouri.

Subject: Relation of Utilities to the Public

Price: $2.75

THURSDAY AFTERNOON - NOVEMBER 10

2:00 PM to 4:00 PM

Presiding: Jack F. Daily, President, Kansas City Section, ASCE; Partner, Lutz & May, Consulting Engineers, Kansas City, Missouri.

Panel Discussion: Which Shall It Be, Indoor or Outdoor Plant?

L. D. Boswell, Principal Engineer, Burns & McDonnell Engineering Company, Kansas City, Missouri.

J. A. Keeth, Vice President, Power Development, Kansas City Power & Light Company, Kansas City, Missouri.

R. D. Woodson, Partner, Black & Veatch, Consulting Engineers, Kansas City, Missouri.

What's the Future of Hydro-Electric Development?

Francis L. Adams, Chief, Bureau of Power, Federal Power Commission, Washington, D. C.

THURSDAY AFTERNOON - NOVEMBER 10

SOCIAL HOUR - 5:30 PM

This event of the local section conference is an annual highlight. Over free cocktails and hors d'oeuvres, conferees have an opportunity to renew old acquaintances and meet new friends.

FRIDAY MORNING - NOVEMBER 11

8:30 AM to 11:45 AM

Presiding: Donald T. McPhee, Manager of Production, Kansas City Power & Light Company, Kansas City, Missouri.

How Does the Civil Engineer Fit Into Atomic Power Generation?

R. N. Bergstrom, Chairman of the Committee on Nuclear Energy for the Power Division of ASCE; Sargent & Lundy, Engineers, Chicago, Illinois.

How Are Power Projects Financed?

W. L. Patterson, Partner, Black & Veatch, Consulting Engineers, Kansas City, Missouri.

What Are the Latest Trends in Stream Plants?

J. H. Harlow, Chief Mechanical Engineer, Philadelphia Electric Company.

FRIDAY LUNCHEON - NOVEMBER 11

12:00 Noon
Georgian Room - Fifth Floor

Speaker: Brig. Gen. A. W. Betts, Director, Advance Research Projects Agency, Office of the Secretary of Defense, Washington, D. C.

Subject: Making Power With Rockets

Price: $2.75

FRIDAY AFTERNOON - NOVEMBER 11

2:00 PM
POWER PLANT TOUR

The committee has arranged an interesting tour of recent power plant improvements in the Kansas City area.

The Conference Committee is headed by Ray E. Lawrence, as Chairman, assisted by the following as sub-committee chairmen: Henry H. Benjes, Program; James B. Smith, Arrangements; Allen H. Wymore, Finance; Leon Boyce, Reception; Bernard M. Nagle, Registration; and Robert O. Drange, Publicity.

FUTURE NEWSLETTERS

News of interest from local sections or individual engineers will be welcomed for use in future Newsletters.

Robert A. Sutherland
Newsletter Editor

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